



U.S. Department of Housing and Urban Development
Office of Policy Development and Research



Testing and Design of Lintels Using Insulating Concrete Forms

PATH (Partnership for Advanced Technology in Housing) is a new private/public effort to develop, demonstrate, and gain widespread market acceptance for the “Next Generation” of American housing. Through the use of new or innovative technologies the goal of PATH is to improve the quality, durability, environmental efficiency, and affordability of tomorrow’s homes.

Initiated at the request of the White House, PATH is managed and supported by the Department of Housing and Urban Development (HUD). In addition, all Federal Agencies that engage in housing research and technology development are PATH Partners, including the Departments of Energy and Commerce, as well as the Environmental Protection Agency (EPA) and the Federal Emergency Management Agency (FEMA). State and local governments and other participants from the public sector are also partners in PATH. Product manufacturers, home builders, insurance companies, and lenders represent private industry in the PATH Partnership.

To learn more about PATH, please contact:



www.pathnet.org

Suite B133
451 7th Street, SW
Washington, DC 20410
202-708-4250 (fax)
e-mail: pathnet@pathnet.org
website: www.pathnet.org

Testing and Design of Lintels Using Insulating Concrete Forms

Prepared for:

The U.S. Department of Housing and Urban
Development
Office of Policy Development and Research
Washington, DC

Portland Cement Association
Skokie, IL

National Association of Home Builders
Washington, DC

Prepared by:

NAHB Research Center, Inc.
Upper Marlboro, MD

Contract H-21172CA

March 2000

Acknowledgments

The NAHB Research Center, Inc., expresses great appreciation to the sponsors of this work in view of its relevance to more affordable design and construction of residential buildings using insulating concrete forms. William Freeborne of the U.S. Department of Housing and Urban Development, Office of Policy Development and Research, is also recognized for his contribution to this program. Special recognition is given to the following manufacturers who provided materials for tests conducted in this project:

American PolySteel Forms, Albuquerque, NM
Lite-Form, Inc., Sioux City, IA
Reddi-Form, Inc., Oakland, NJ

The Research Center personnel involved in this work include:

Shawn McKee, Principal Investigator
Jay Crandell, P.E., Technical Reviewer
Andrea Vrankar, P.E., R.A., Project Manager
Ricardo Hicks, Laboratory Support
Christian Jacobs, Laboratory Support
Lisa Zimmerman, Administrative Support

Disclaimer

The work that provided the basis for this publication was supported by funding under a grant with the U.S. Department of Housing and Urban Development. The substance and findings of the work are dedicated to the public. The author and publisher are solely responsible for the accuracy of the statements and interpretations contained in this publication. Such interpretations do not necessarily reflect the views of the Government.

The NAHB Research Center, Inc. is a not-for-profit subsidiary of the National Association of Home Builders (NAHB). The NAHB has 190,000 members, including 50,000 builders who build more than 80 percent of new American homes. NAHB Research Center conducts research, analysis, and demonstration programs in all areas relating to home building, and carries out extensive programs of information dissemination and interchange among members of the industry and between the industry and the public.

Foreword

The increasing complexity of homes, the use of innovative materials and technologies, and the increased population in high-hazard areas of the United States have introduced many challenges to the building industry and design professional. These challenges call for the development and continual improvement of efficient engineering methods for housing applications as well as for education of designers in the uniqueness of housing as a structural design problem.

This report summarizes the results from the test program focusing on the structural performance of concrete lintels without shear reinforcement. The performance (i.e. strength and ductility) of simply-supported concrete beams without shear reinforcement and with minimal tensile reinforcing steel subjected to third point loading is evaluated. Design recommendations for concrete lintel or beam design for each of the major ICF systems that is also presented.

Susan M. Wachter
Assistant Secretary for Policy Development and Research

Table of Contents

| | |
|--|----|
| Introduction..... | 1 |
| Background..... | 2 |
| ACI 318-99 Design Provisions | 5 |
| Conventional ICF Lintel (Concrete Beam) Design | 7 |
| Experimental Program | 10 |
| Test Procedure | 14 |
| Test Results..... | 16 |
| Design Evaluation..... | 20 |
| Design Recommendations | 29 |
| Conclusions..... | 30 |
| Recommendations..... | 31 |
| References..... | 31 |

List of Tables

| | |
|--|----|
| Table 1: Specimen Specifications..... | 10 |
| Table 2: Concrete Mix Data..... | 12 |
| Table 3: Concrete Compression Tests | 13 |
| Table 4: Rebar Tension Tests | 13 |
| Table 5: Results of ICF Lintel Tests..... | 16 |
| Table 6: Comparison of Tensile Reinforcing Steel of Flat Lintel Specimens to ACI 318-99 Minimum Requirements..... | 21 |
| Table 7: Tested vs. Predicted Shear and Moment Capacities Based on ACI 318-99 For the Flat ICF Lintels | 22 |
| Table 8: Comparison of Tensile Reinforcing Steel of Waffle-Grid Lintel Specimens to ACI 318-99 Minimum Requirements and to Balanced Conditions | 24 |
| Table 9: Tested vs. Predicted Shear and Moment Capacities Based on ACI 318-99 For the Waffle-Grid ICF Lintels | 25 |
| Table 10: Equivalent Web Thickness for Waffle-Grid Specimens | 26 |
| Table 11: Comparison of Tensile Reinforcing Steel of Screen-Grid Lintel Specimens to ACI 318-99 Minimum Requirements and to Balanced Conditions | 27 |
| Table 12: Tested vs. Predicted Shear and Moment Capacities Based on ACI 318-99 For the Screen-Grid ICF Lintels..... | 28 |
| Table 13: Equivalent Web Thickness for Screen-Grid Specimens..... | 28 |

List of Figures

| | |
|--|----|
| Figure 1: Diagrams and Dimensions of ICF Systems Used to Construct the Lintel Specimens..... | 11 |
| Figure 2: Flat 8x24 Specimen in Universal Test Machine | 14 |
| Figure 3: Screen-grid 6x24 Specimen in Universal Test Machine..... | 15 |
| Figure 4: Steel I-Beam, Loading Points, and Deflectometer..... | 15 |
| Figure 5a: Typical Failure Mode for the Long-Span ICF Flat Lintel (4x12 specimen after excessive deflection and tensile steel yielding) | 17 |
| Figure 5b: Typical Failure Mode for the Long-Span ICF Flat Lintel (4x24 specimen after excessive deflection and tensile steel yielding) | 17 |
| Figure 5c: Typical Failure Mode for the Long-Span ICF Flat Lintel (4x24 specimen with foam removed at primary failure location)..... | 18 |
| Figure 6: Typical Failure Mode for the Long-Span ICF Waffle-Grid Lintel | 18 |
| Figure 7a: Typical Failure Mode for the Long-Span ICF Screen-Grid Lintel (6x12 specimen) | 19 |
| Figure 7b: Typical Failure Mode for the Long-Span ICF Screen-Grid Lintel (6x24 specimen with foam removed from primary failure location) | 20 |
| Figure 8: The Influence of the Reinforcement Ratio on the Predicted Shear Capacity for the Short-Span Flat ICF Lintel Specimens | 23 |

Introduction

Concrete lintels are used in Insulating Concrete Form (ICF) construction to transfer loads above window and door openings in walls. Current design methods typically require vertical shear reinforcement as well as horizontal tensile reinforcing steel to support bending loads. However, in many residential and light-construction situations, shear reinforcement may not be necessary to achieve adequate performance. Shear reinforcement is difficult to place in ICF forms, complicates the construction process, and creates difficulties in placing the concrete. Therefore, this study is intended to investigate the need for shear reinforcement and, if possible, develop an improved ICF lintel design methodology based on testing results.

In May 1998, initial experimental studies were reported on various short-span configurations of ICF lintels [1]. The purpose of the testing was to determine if more economical designs, i.e. eliminating shear reinforcement, adversely affected the performance of concrete lintels. According to the *Building Code Requirements for Reinforced Concrete (ACI 318-99)* [2], shear reinforcement was required for all ICF lintels in the *Prescriptive Method for Insulating Concrete Forms in Residential Construction (Prescriptive Method)* [3]. In all cases the tested capacity of the ICF lintels without shear reinforcement outperformed the *ACI 318-99* predicted capacities. However, a general design approach was not developed since lintels with longer-spans were not considered.

Expanding upon the previous work, this test program focuses on the structural performance of longer-span concrete lintels without shear reinforcement. This report will evaluate the structural performance of ICF lintels that span approximately 12 ft (3.7 m) and revisit the results from the May 1998 testing program. Due to the longer-span lengths and minimal tensile reinforcing steel amounts, yielding under a bending load is expected to result in a predictable ductile behavior prior to shear failure for the longer-span ICF lintels.

The objectives of the testing and evaluation program are as follows:

- (1) Verify the performance (i.e. strength and ductility) of longer-span, simply-supported beams without shear reinforcement and with minimal tensile reinforcing steel (i.e. less than the minimum tensile steel ratio required by *ACI-318*);
- (2) Verify and improve methods of calculating the bending capacity of ICF lintels;
- (3) Verify and improve methods of calculating shear capacity of ICF lintels that have tensile reinforcing steel, but no shear reinforcement; and,
- (4) Develop design recommendations for concrete lintel or beam design for each of the major ICF systems that is reasonably simple, safe, and accurate.

Background

Shear failure of reinforced concrete, more properly called diagonal tension failure is a failure under combined shearing force and bending moment. Such failures reduce the strength of structural elements below the flexural capacity and considerably reduce the ductility of the elements. Especially for the latter reason, shear failures are generally considered undesirable [4]. In spite of many decades of experimental research and the use of highly sophisticated analytical tools, shear failure is not yet fully understood.

For under reinforced beams, flexural failure is initiated by gradual yielding of the tensile steel, accompanied by cracking of the concrete at large deflections, giving ample warning and providing the opportunity for load distribution or corrective measures. Conversely, if a beam without sufficient shear capacity is overloaded to failure, shear collapse is likely to occur suddenly, with no advance warning of distress. Because of these differences in behavior, shear reinforcement is usually required in reinforced concrete beams to ensure that flexural yielding occurs before shear failure [5].

Shear is transmitted from one plane to another in various ways in reinforced concrete members. The behavior, including the failure modes, depends on the method of shear transmission. The main types of shear transfer are the following: (a) shear stress in the uncracked concrete; (b) shear friction; (c) dowel action; (d) arch action; and (e) shear reinforcement [4].

The simplest method of shear transfer is by shearing stresses. This occurs in uncracked members or in the uncracked portions of structural members. The interaction of shear stresses with tensile and compressive stresses produces principal stresses that may cause inclined cracking or a crushing failure of the concrete [4].

Shear friction is the transfer mechanism in which shear is transferred across a definite plane or surface where slip may occur. The basic approach is to assume that the concrete may crack in an unfavorable manner, or that slip may occur along a predetermined plane of weakness. Reinforcement must be provided crossing the potential or actual crack or shear plane to prevent direct shear failure [5].

If reinforcing bars cross a crack, a doweling force in the bar will resist shearing displacements along the crack. The dowel force gives rise to tensions in the surrounding concrete and this in combination with the wedging action of the bar deformations produce splitting cracks along the reinforcement. This in turn decreases the stiffness of the concrete around the bar and therefore the dowel force. Relative to other shear transfer mechanisms, the dowel shear force is generally not dominant in beams [4]. In beams, splitting cracks develop along the tension reinforcement at inclined cracks as a result of the dowel effects. This allows the inclined cracks to open, which in turn reduces the shear friction along the diagonal crack and thus leads to failure.

In deep beams part of the load is transmitted to the supports by arch action. Arch action permits the transfer of a vertical concentrated force to a reaction in a deep member and thereby reduces the contribution of the other types of shear transfer. For arch action to develop, a horizontal reaction component is required at the base of the arch. In beams this is usually provided by the tie action of the longitudinal bars [4].

Shear reinforcement directly increases the shear capacity, but also aids the several kinds of shear transfer action. Shear reinforcement restricts the widening of inclined cracks in beams and thus increases or maintains the shear friction, dowel action, and arch action [4].

Shear failures of beams are characterized by the occurrence of inclined cracks. In some cases inclined cracking is immediately followed by member failure and in other cases, the inclined cracks are stabilized and substantially more shear force may be applied before the member fails. Inclined cracks in the web of the beam may develop either before a flexural crack occurs in their vicinity (web-shear crack) or as an extension of a previously developed flexural crack (flexural-shear crack) [5].

An evaluation of 194 beam tests was conducted to investigate a wide variety of concrete beam configurations [6]. It was found that in regions with large shear and small moment, diagonal tension cracks (web-shear cracks) form at an average or nominal shear stress of approximately

$$v_{cr} = \frac{V_{cr}}{b_w d} = 3.5\sqrt{f'_c} \quad (\text{Equation 1})$$

where V_{cr} is the shear force at which the formation of the crack was observed. Web-shear cracking is relatively rare and occurs chiefly near supports of deep, thin-webbed beams or at inflection points of continuous beams [6].

The evaluation of the aforementioned 194 beam tests also indicates that in the presence of large moments the nominal shear stress at which diagonal tension cracks (flexural-shear cracks) form and propagate is conservatively given by [6]

$$v_{cr} = \frac{V_{cr}}{b_w d} = 1.9\sqrt{f'_c} \quad (\text{Equation 2})$$

Comparison of Equation 1 and Equation 2 shows that large bending moments can reduce the shear force at which diagonal tension cracks form to roughly one-half the value at which they would form if the moment is zero or nearly so. It is evident that the shear at which diagonal tension cracks develop depends on the ratio of shear force to the bending moment.

Recent research studies indicate that Equation 2 over-emphasizes the influence of f'_c and does not recognize the important influence of the tensile reinforcing steel ratio (ρ_w) and the span-to-depth ratio (l_n/h) [4][7]. For example, the most significant conclusion from the 133 beams tested by Kani was that the shear strength of rectangular, reinforced concrete beams does not strongly depend on concrete strength [7]. He determined that the tensile reinforcing steel ratio (ρ_w) and the span-to-depth ratio (l_n/h) had the greatest impact on the shear strength.

ACI Committee 326 conducted a systematic study of data from more than 440 tests and determined that the shear capacity depends on three variables [6]:

- (1) The dimensionless quantity M/Vd involving the bending moment M , the shear force V , and the effective depth d (this value was developed from the span-to-depth ratio, a/d , for simple beams with a center point load or with two symmetrical point loads, where a is the length of the shear span or the distance from the support to the nearest point load and d is the effective depth)
- (2) The longitudinal reinforcement ratio, ρ_w
- (3) The concrete compressive strength, f'_c

Apart from the aforementioned influence of M/Vd , increasing values of the tensile reinforcing steel ratio have a beneficial effect in that they increase the shear at which diagonal cracks develop. This is because larger amounts of longitudinal steel result in smaller and narrower tension cracks prior to the formation of diagonal cracking, and leaving a larger area of uncracked concrete available to resist shear.

Two straight lines corresponding to the following expression conservatively defines the trend of the test data

$$v_{cr} = \frac{V_{cr}}{bd} = 1.9\sqrt{f'_c} + 2500 \frac{\rho_w Vd}{M} \leq 3.5\sqrt{f'_c} \quad (\text{Equation 3})$$

where $\rho_w = A_s/b_w d$ and 2,500 is an empirical constant in pounds per square inch [6]. The ACI Committee 326 choice of lines was influenced by two major considerations. First, the equation should be simple to facilitate everyday design work, and second, the equation should be such that the ultimate strength of beams resulting from practical design will be governed by flexure rather than shear [6]. To satisfy the first consideration, a simple straight-line representation was chosen versus a more complex yet accurate equation. To satisfy the second consideration, the lines were placed near the lower extremes of observed shear stress rather than as average values. Equation 3 was chosen such that less than 30 percent of the data fell below the estimate. The data was fit to a normal distribution and the coefficient of variation that characterized the data was 15.1 percent[6].

Some concrete members have depths much greater than normal in relation to their span. The main loads and reactions act in the plane of the member. Members of this type are called deep beams. They can be defined as beams having a span-to-depth ratio, l_n/h , of about 5 or less, or having a shear span a less than about twice the depth. The behavior of deep beams is significantly different from that of beams of more normal proportions, requiring special consideration in analysis, design and detailing of reinforcement. Because of their proportions, deep beams are likely to have strength controlled by shear. On the other hand, their strength is likely to be significantly greater than predicted by the usual equations due to an arching effect.

Because of the geometry of deep beams, they behave as two-dimensional rather than one-dimensional members and are subjected to a two-dimensional state of stress. Stresses in deep beams before cracking can be studied using the methods of two-dimensional elasticity or finite element analysis. Such studies confirm that the usual hypothesis, that plane sections before

bending remain plane after bending, does not hold for deep beams. Significant warping of the cross sections occurs because of high shear stresses. The resulting strain distribution is no longer considered as linear, and shear deformations that are neglected in normal beams become significant compared to pure flexure. Consequently, the stress block becomes non-linear even in the elastic range. At the limit state of ultimate load, the compressive stress distribution in the concrete no longer follows the same parabolic shape or intensity as that used for normal beams.

Shear strength of deep beams may be as much as 2 or 3 times greater than that predicted using conventional equations developed for members of normal proportions (i.e., Equations 1 to 3). A significant portion of the load for deep beams is transferred directly from the point of application to the supports by diagonal compression struts. Diagonal cracks that form roughly in a direction parallel to a line from the load to support isolate the compression strut, which acts with the horizontal compression in the concrete and tension in the main reinforcement to equilibrate loads. The geometry of this mechanism and relative importance of each contribution to shear strength clearly depends on the proportions of the member as well as the placement of the loads and reactions.

Because of the orientation of the principal stresses in deep beams, when diagonal cracking occurs, it will be at a slope greater than 45 degrees in most cases. Consequently, while it is important to include vertical stirrups, they are apt to be less effective than horizontal web reinforcement. The horizontal bars are more effective because they act more in the direction perpendicular to the diagonal crack, thus improving shear transfer by aggregate interlock, and also because horizontal web reinforcement contributes to the shear transfer by dowel action.

ACI 318-99 Design Provisions

The design of bending members for shear is based on the assumption that the concrete resists part of the shear, and any load exceeding what the concrete is capable of resisting has to be resisted by shear reinforcement. The basic rationale for the design of shear reinforcement is to provide steel that bridges across the diagonal tension cracks. Diagonal tension cracks are indicative of an impending shear failure in concrete beams and lintels. Shear reinforcement, commonly known as stirrups in concrete beams, prevents the diagonal tension cracks from propagating across the depth of the member and provides for continued capacity subsequent to concrete cracking.

Shear strength, V_c , can be calculated using either a simplified or more detailed calculation in *ACI 318-99*. For members that are subject to shear and flexure only, the amount of shear force that the concrete alone, unreinforced for shear, can resist according to the simplified method is

$$V_c = 2\sqrt{f'_c}b_w d \quad \text{ACI 318-99 Equation 11-3}$$

The more detailed calculation computes the amount of shear force that the concrete alone can resist according to

$$V_c = \left(1.9\sqrt{f'_c} + 2500\rho_w \frac{Vd}{M} \right) b_w d \leq 3.5\sqrt{f'_c}b_w d \quad \text{ACI 318-99 Equation 11-5}$$

where,

- f'_c = compressive strength of concrete (psi)
- b_w = web width, equivalent to b for rectangular beams (in)
- d = effective depth (in)
- ρ_w = tensile reinforcing steel ratio (dimensionless)
- V = shear force at section considered (lb)
- M = bending moment occurring simultaneously with V (in-lb)
- Vd/M = shall not be greater than 1.0 (dimensionless)

Theoretically, no web reinforcement should be required when the following condition is met:

$$V_u \leq \phi V_c$$

where,

- V_u = shear force induced by factored design loads
- ϕ = strength reduction factor of 0.85 required by *ACI 318-99*
- V_c = nominal concrete shear strength (*ACI 318-99* Equation 11-3 or Equation 11-5)

According to the *ACI 318-99* commentary, the formation of inclined cracks may lead to sudden failure in unreinforced concrete beams. Therefore, the ACI Code requires that a minimum area of shear reinforcement be provided in reinforced concrete flexural members except where V_u is less than $\phi V_c/2$. This requirement is intended to provide assurance that a ductile failure (i.e. yielding of the tensile reinforcing steel) will be experienced in beams without shear reinforcement rather than an abrupt shear failure. When required, shear reinforcement is designed to resist the shear force to also insure that a ductile failure is experienced due to yielding of tensile reinforcing steel. When $\phi V_c/2 \leq V_u \leq \phi V_c$ then a minimum area of shear reinforcement required is determined using the following equation:

$$A_{v \min} = 50 \frac{b_w s}{f_y} \quad \text{ACI 318-99 Equation 11-14}$$

where,

- $A_{v \min}$ = total cross sectional area of shear reinforcement within a distance s (in²)
- b_w = web width, or b for rectangular beams (in)
- s = center-to-center spacing of shear reinforcement in a direction parallel to the longitudinal reinforcement (in)
- f_y = yield strength of shear reinforcement steel (psi)

According to *ACI 318 – 99* Section 11.8, special provisions for shear are to be applied to beams for which l_w/h is less than 5 and which are loaded on one face and supported on the opposite face so that diagonal compression struts can form between the loads and the supports. The special shear provisions apply to simply supported deep beams but not to fixed end or continuous deep beams. In a fixed end or continuous beam, the critical section for shear occurs at a point where M approaches zero. As a result the second term in *ACI 318* Equation 11-29 becomes large. For this reason fixed end and continuous beams are to be designed for shear according to the normal beam design procedures.

Because of the strength increase attainable for deep beams owing to strut and tie action, the ACI Code provisions permit the usual value of the concrete shear strength V_c , calculated by ACI 318-99 Equation 11-5 to be increased by a multiplier that depends on M/Vd . For deep beams, the concrete contribution to shear strength can be computed from

$$V_c = \left(3.5 - 2.5 \frac{M}{Vd} \right) \left(1.9 \sqrt{f'_c} + 2500 \rho_w \frac{Vd}{M} \right) b_w d \leq 3.5 \sqrt{f'_c} b_w d \quad \text{ACI 318-99 Equation 11-29}$$

with the restrictions that the multiplier $(3.5 - 2.5M/Vd)$ must not exceed 2.5 and that V_c must not exceed $6\sqrt{f'_c} b_w d$. For deep beams, shear reinforcement is not required when V_u is less than ϕV_c . However, when shear reinforcement is required, it shall be provided throughout the span.

Virtually all provisions in *ACI 318-99* regarding shear are intended to prevent failures resulting from diagonal tension rather than direct shear transfer [5]. The shear friction provisions provide design methods for conditions where shear transfer should be considered. The shear friction concept assumes that an unfavorable crack will form and the reinforcement that is provided across the crack will resist relative displacement along it [5]. When shear acts along the crack, one crack face slips relative to the other. This slip is accompanied by separation of the rough crack faces. As the separation increases stress is realized in the reinforcement crossing the crack.

When reinforcement is perpendicular to the shear plane, the shear strength, V_n , is computed by

$$V_n = A_{vf} f_y \mu \quad \text{ACI 318-99 Equation 11-25}$$

and when the shear friction reinforcement is inclined to the shear plane, such that shear force produces tension in the shear friction reinforcement, shear strength, V_n , is computed by

$$V_n = A_{vf} f_y (\mu \sin \alpha_f + \cos \alpha_f) \quad \text{ACI 318-99 Equation 11-26}$$

where

- A_{vf} = area of shear friction reinforcement (in²)
- f_y = yield strength of shear friction reinforcement steel (psi)
- μ = coefficient of friction (dimensionless)
- α_f = angle between shear friction reinforcement and shear plane

Conventional ICF Lintel (Concrete Beam) Design

According to the *Structural Design of Insulating Concrete Form Walls in Residential Construction* [9], ICF lintels are designed for bending using the equations below in accordance with ACI Chapter 10, "Flexure and Axial Loads". The amount of tensile reinforcing steel is limited such that it will yield before the concrete reaches its ultimate compressive strength as required by *ACI 318-99*.

$$M_u \leq \phi M_n \quad (\text{Equation 4})$$

$$\phi M_n \leq \phi A_s f_y \left(d - \frac{a}{2} \right) \quad (\text{Equation 5})$$

$$a = \frac{A_s f_y}{0.85 f_c ' b} \quad (\text{Equation 6})$$

where,

- ϕ = the strength reduction factor of 0.9 for flexure per ACI 9.3.2
- a = depth of equivalent rectangular stress block (in)
- A_s = area of tensile reinforcing steel (in²)
- b = width of compression face of member (in)
- d = distance from extreme compression zone (top of beam) to centroid of tensile reinforcement (in)
- f'_c = specified compressive strength of concrete (psi)
- f_y = specified yield strength of reinforcement (psi)
- M_n = nominal moment strength (in-lb)
- M_u = moment induced by factored design loads (in-lb)

Regardless of the design conditions, *ACI 318-99* requires a minimum amount of tensile reinforcing steel determined by the greater of the following equations:

$$A_{s,\min} = \frac{3\sqrt{f'_c} b_w d}{f_y} \geq \frac{200 b_w d}{f_y} \quad \text{ACI 318-99 Equation 10-3}$$

where the variables are as previously defined. The minimum tensile steel requirement outlined by ACI Equation 10-3 need not be applied if the area of tensile reinforcing steel provided is at least one-third greater than that required by analysis.

The lintel design approach used in the *Prescriptive Method* followed the conventional design method outlined above with the following exceptions:

- (1) Shear reinforcement was not required for ICF lintels with spans of 4 ft (1.22 m) or less based on previous testing [1];
- (2) The shear capacity of the lintels were analyzed as normal beams regardless of the span-to-depth ratio, l_n/h , and
- (3) Tensile reinforcing steel amounts were permitted to be less than minimum required by *ACI 318-99*.

The technical and practical reasons for allowing less tensile reinforcing steel than required by *ACI 318-99* was as follows:

- (1) The design capacity required in the application to residential load conditions did not require the minimum amount of tensile reinforcing steel dictated in *ACI 318-99*.
- (2) Providing more reinforcement and capacity than required will adversely impact constructability of ICF structures by interfering with concrete placement in the forms.
- (3) The overdesign, resulting from adherence to *ACI 318-99* minimum tensile reinforcing steel requirement, would result in unnecessary cost impacts that deter from the effective use of ICF systems in residential construction.

The intent of this study is to provide further confirmation of the modifications to the conventional design practice of *ACI 318-99* and to develop improved lintel design methods appropriate for use with ICF systems. In fact, the lack of design methodology for the screen-grid ICF system (non-solid concrete cross section) prevented this system from being used as lintels in the *Prescriptive Method* [1].

Experimental Program

A total of 11 long-span ICF lintels using three different ICF systems were constructed and tested to evaluate their structural performance without the use of shear reinforcement. An overview of the long-span ICF lintel specimens is given in Table 1. A description of the 18 short-span ICF lintel specimens from the previous testing is also included in Table 1. Diagrams showing the shape and dimensions of the individual ICF blocks used to form the lintels are shown in Figure 1.

Table 1
Specimen Specifications

| Test Specimen | Width, b_w (in) | Depth, $d^{(1)}$ (in) | Span ⁽²⁾ (in) | Tension Reinforcement |
|---|--------------------|-----------------------|--------------------------|-----------------------|
| Long-Span ICF Lintel Specimens (current testing) | | | | |
| FLAT1_8x24 | 8 | 21.75 | 136 | 1 - #4 |
| FLAT2_4x12 | 4 | 10 | 133 | 1 - #4 |
| FLAT3_8x12 | 8 | 10 | 133 | 1 - #4 |
| FLAT4_4x24 | 4 | 22 | 137.75 | 1 - #4 |
| WAFFLE1_8x16 | 2 | 13.5 | 133 | 1 - #4 |
| WAFFLE2_8x16 | 2 | 14 | 132 | 1 - #4 |
| WAFFLE3_8x16 | 2 | 14 | 132 | 2 - #4 |
| WAFFLE4_8x8 | 2 | 6 | 131 | 1 - #4 |
| SCREEN1_6x12 | N/A ⁽⁴⁾ | 10 | 134 | 1 - #4 |
| SCREEN2_6x12 | N/A ⁽⁴⁾ | 10 | 134 | 1 - #4 |
| SCREEN3_6x24 | N/A ⁽⁴⁾ | 21.5 | 134 | 1 - #4 |
| Short-Span ICF Lintel Specimens (previous testing) | | | | |
| FLAT1_4x12 | 4 | 10 | 43 | 1 - #4 |
| FLAT2_4x12 | 4 | 10 | 43 | 1 - #4 |
| FLAT1_4x24 | 4 | 22 | 43 | 1 - #4 |
| FLAT1_8x12 | 8 | 10 | 43 | 1 - #4 |
| FLAT2_8x12 | 8 | 10 | 43 | 1 - #4 |
| FLAT1_8x24 | 8 | 22 | 43 | 1 - #4 |
| FLAT1_4x12a | 4 | 10 | 40 | 2 - #5 |
| FLAT1_8x12a | 8 | 10 | 40 | 2 - #5 |
| WAFFLE1_6x8 | 2 | 6 | 36 | 1 - #4 |
| WAFFLE2_6x8 | 2 | 6 | 36 | 1 - #4 |
| WAFFLE1_6x16 | 2 | 14 | 36 | 1 - #4 |
| WAFFLE2_6x16 | 2 | 14 | 36 | 1 - #4 |
| WAFFLE1_8x16 | 2 | 13.75 | 36 | 1 - #4 |
| WAFFLE2_8x16 | 2 | 13.75 | 36 | 1 - #4 |
| SCREEN1_6x12 | N/A ⁽³⁾ | 10 | 38 | 1 - #4 |
| SCREEN2_6x12 | N/A ⁽³⁾ | 10 | 38 | 1 - #4 |
| SCREEN1_6x24 | N/A ⁽³⁾ | 22 | 38 | 1 - #4 |
| SCREEN2_6x24 | N/A ⁽³⁾ | 22 | 38 | 1 - #4 |

For SI: 1 in = 25.4 mm.

Notes: 1. Depth is measured from the top of the beam to the centroid of the tension reinforcement rounded to nearest ¼ inch.

2. Span is measured from center to center of end bearing supports and is rounded to the nearest ¼ inch.

3. ACI 318-99 does not provide a method to analyze beam cross sections with voids.

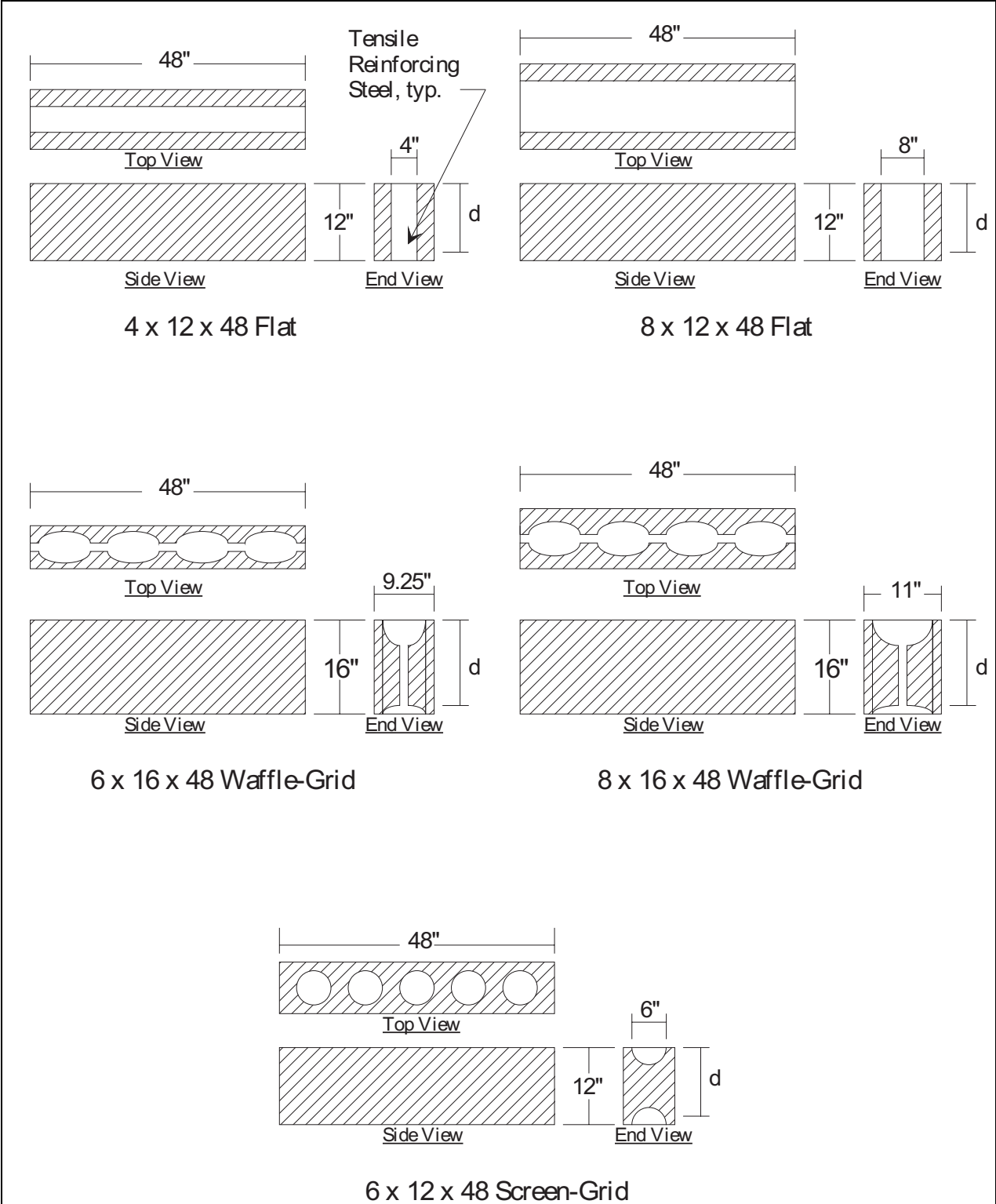


Figure 1
Diagrams and Dimensions of ICF Systems Used
to Construct the Lintel Specimens
(48 in specimens = 1 block, 144 in specimens = 3 connected blocks)

Different manufacturers provided the three types of ICFs. The 11 lintels were cast off the ground and enclosed with oriented strand board (OSB) on both ends and the bottom. The ICF systems utilize the integral form ties to support the horizontal reinforcement. Holes were also cut at the appropriate height in the OSB end panels to support the ends of the horizontal (i.e. tensile) reinforcement. Shear (i.e. vertical) reinforcement was not used in any of the specimens.

The concrete mix used for all of the test specimens complied with the *Prescriptive Method* [3]. The mix was designed to provide a minimum 28-day compressive strength of 2,500 psi (17.2 MPa). Quantities of materials incorporated in the selected concrete mix are shown in Table 2. The concrete was supplied by a local ready-mix company and, after certain adjustments were made to optimize workability, had a measured on-site slump of 6 in (152 mm) according to ASTM C143 [10].

Table 2
Concrete Mix Data

| Mix Ingredient | Quantity |
|------------------------------|----------|
| Cement Type I/II | 2,115 lb |
| Concrete Sand | 6,541 lb |
| #67 Washed Gravel | 9,000 lb |
| Daravair 1000 ¹ | 20 oz |
| WRDA with HYCOL ² | 60 oz |
| Water | 150 gal |

For SI: 1 lb = 4.45 N, 1 oz = 0.28 N, 1 gal = 3.79 l

Notes:

1. Daravair 1000 is an air-entraining admixture and is formulated to comply with Specification for Air-Entraining Admixtures for Concrete, ASTM Designation C 260.
2. WRDA with HYCOL is a water reducing admixture and is formulated to comply with Specification for Chemical Admixtures for Concrete, ASTM Designation C 194.

Concrete was placed in the forms in layers (lifts) of a depth equal to approximately 1 ft (0.30m). Each lift was manually consolidated using a 0.5 in (12.7 mm) diameter steel rod. The total time duration of the cast was approximately 1.5 hours for all specimens. The form insulation was removed either prior to testing or after testing in the failure region. Although localized “honeycombing” was evident in a few short-span specimens from the previous test program, only one of the 11 long-span specimens displayed any evidence of this type of construction flaw in the current study. It should be noted that the “honeycombing” observed was minor and did not affect the structural performance. Additionally, ten of the eleven specimens used only one #4 tensile reinforcing bar located approximately 2 in (50.8 mm) from the bottom of the form, which helped to ensure uniform flow of concrete around the reinforcement.

Cylindrical concrete specimens were also cast following ASTM C39 [11], ASTM C31 [12], and ASTM C192 [13]. Twenty 6 in x 12 in (152.4 mm x 304.8 mm) cylinders were cast to represent the concrete incorporated in the test specimens. After 48 hours the cylinders were split into two batches. The first batch was moist cured, while the second batch was field cured. The cylinders were tested at 7, 28 and 56 days to determine the compressive strength of the concrete. Three of the field-cured cylinders were also tested after 605 days, which provided compressive strength data for use with this study of longer-span specimens. Table 3 summarizes the results.

Table 3

Concrete Compression Tests

| Test Specimens | 7 day (psi) | 28 day (psi) | 56 day (psi) | 605 day (psi) |
|------------------|--------------|--------------|--------------|---------------|
| Laboratory-Cured | 2,252 | 2,701 | 2,775 | - |
| | 2,309 | 2,692 | 2,829 | - |
| | 2,091 | 2,887 | 2,891 | - |
| AVERAGE | 2,217 | 2,760 | 2,831 | - |
| COV | 0.051 | 0.040 | 0.020 | - |
| Field-Cured | 2,090 | 2,625 | 2,690 | 2,923 |
| | 2,160 | 2,614 | 2,710 | 3,151 |
| | 2,050 | 2,619 | 2,640 | 2,791 |
| AVERAGE | 2,100 | 2,619 | 2,680 | 2,955 |
| COV | 0.027 | 0.002 | 0.013 | 0.062 |

For SI: 1 psi = 6.90 kPa.

According to *ACI 318-99*, the compressive strength, f'_c , should be determined from the laboratory-cured specimens. The field-cured specimens are usually used to check the adequacy of curing and protection of concrete exposed to field conditions. However, no laboratory-cured specimens remained after 605 days. Only field-cured cylinders were available to determine the compressive strength. The previous compressive strength testing consistently showed field-cured specimens were approximately 5.5 percent lower in compressive strength at 7 days, 28 days, and 56 days compared to the laboratory-cured specimens. However, the additional long span lintels tested under this study were stored outdoors with the concrete cylinders for the entire 605 days. Therefore, the field-cured specimens tested at 605 days provide a reasonable characterization of the compressive strength of the lintels tested in this report. An average value of 2,955 psi (20.4 MPa) for the 605 day field cure time was used as the concrete compression strength (f'_c). This average is representative of the concrete compressive strength during the testing period of this study. An average value of 2,795 psi (19.3MPa) was used in the previous study since the short span lintel tests were conducted between 28 days and 56 days.

Grade 40 tensile reinforcing steel was ordered for fabricating the lintel specimens. Tension tests performed at the University of Maryland revealed yield strengths closer to Grade 60 steel. Table 4 lists the results. The #5 reinforcing bar was used in two short span lintels tested in the previous study [1].

**Table 4
Rebar Tension Tests**

| Test Specimens | f_y (ksi) | Test Specimens | f_y (ksi) |
|----------------|--------------|----------------|--------------|
| #4 Rebar | | #5 Rebar | |
| Specimen 1 | 60.1 | Specimen 1 | 60.1 |
| Specimen 2 | 66.8 | Specimen 2 | 58.0 |
| Specimen 3 | 67.2 | Specimen 3 | 59.1 |
| AVERAGE | 64.7 | AVERAGE | 59.1 |
| COV | 0.062 | COV | 0.018 |

For SI: 1 psi = 6.90 kPa.

Test Procedure

The long-span and short-span lintels were cast at the same time prior to the first phase of testing in October 1997. Tests on the long-span lintel specimens commenced at a concrete age of 605 days and continued over a duration of 20 days. The age of the long-span lintels was the result of the time lag between the two phases (phase 1 – short-span lintels, phase 2 – long-span lintels) of the test program. Each specimen was tested in a manner similar to that described in ASTM C78-94 [14]. The NAHB Research Center's Universal Test Machine (UTM) applied the load, under deflection control, to the specimens until failure at a constant rate between 0.075 in (1.9 mm) to 0.1 in (2.5 mm) per minute. A deflectometer was used to measure the displacement at midspan. Third-point loading was applied to the lintel specimens using a steel I-beam attached to the UTM crosshead. The location of the concentrated load points and reaction bearings were adjusted to coincide with the center of the cores in the waffle-grid and screen-grid ICF lintels. The lintels were simply supported at both reactions on rollers. Leather shims were placed between the bearing plates and the concrete at the load points to minimize the effect of surface roughness. When necessary, an even bearing surface was provided by rasping the concrete surface. Specific details of the test apparatus and setup are shown in Figures 2 through 4.

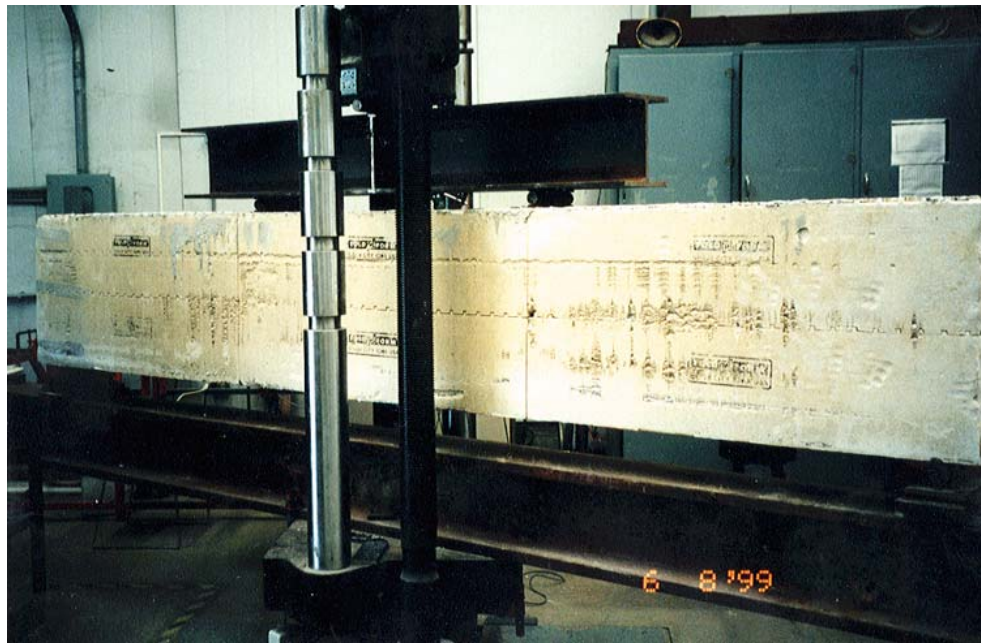


Figure 2
Flat 8x24 Specimen in Universal Test Machine

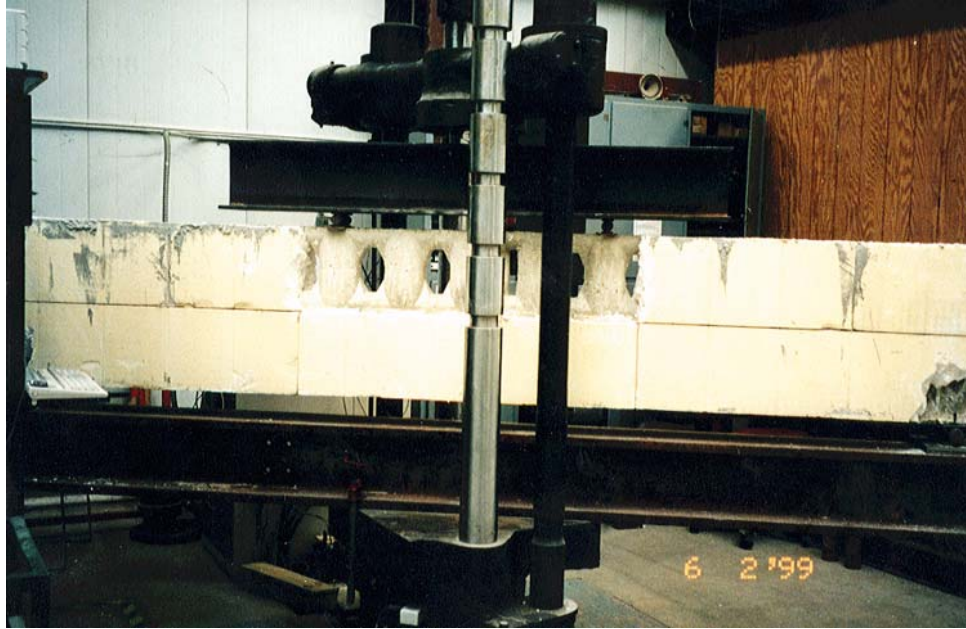


Figure 3
Screen-grid 6x24 Specimen in Universal Test Machine



Figure 4
Steel I-Beam, Loading Points, and Deflectometer

Test Results

A summary of both phases of the ICF lintel testing is given in Table 5. The tested loads are the total two-point load on the beam including the weight of the lintel, the weight of the loading beam, and any pre-load applied before the test was started.

Table 5
Results of ICF Lintel Tests

| Test Specimen | Span (in) | Shear Span ¹ (in) | Effective Depth (d) (in) | Web Width (b) (in) | Tensile Steel | Test Result | | | | |
|---|-----------|------------------------------|--------------------------|--------------------|---------------|---------------------------|-------------------------|---------------------|---------------------------------|------------------------------|
| | | | | | | Failure Mode ² | Ultimate Capacity (lbs) | Load at Yield (lbs) | Δ at Ultimate ³ (in) | Δ at Yield ⁴ (in) |
| Long-Span ICF Lintel Specimens (current testing) | | | | | | | | | | |
| FLAT1_8x24 | 136.00 | 45.00 | 21.75 | 8.00 | 1-#4 | BD | 15,494 | 13,047 | 3.20 | 0.09 |
| FLAT2_4x12 | 133.00 | 41.50 | 10.00 | 4.00 | 1-#4 | BD | 5,960 | 5,630 | 2.68 | 0.46 |
| FLAT3_8x12 | 133.00 | 43.00 | 10.00 | 8.00 | 1-#4 | BD | 7,223 | 6,056 | 2.98 | 0.30 |
| FLAT4_4x24 | 137.75 | 46.00 | 22.00 | 4.00 | 1-#4 | BD | 16,102 | 11,733 | 3.22 | 0.21 |
| WAFFLE1_8x16 | 133.00 | 48.00 | 14.00 | 2.00 | 1-#4 | SB | 10,116 | 6,912 | 2.27 | 0.37 |
| WAFFLE2_8x16 | 132.00 | 47.50 | 13.50 | 2.00 | 1-#4 | SB | 10,522 | 8,736 | 2.73 | 0.34 |
| WAFFLE3_8x16 | 132.00 | 47.75 | 14.00 | 2.00 | 2-#4 | SB | 17,761 | 15,233 | 1.55 | 0.44 |
| WAFFLE4_8x8 | 131.00 | 48.00 | 6.00 | 2.00 | 1-#4 | SB | 3,682 | 3,217 | 2.86 | 0.69 |
| SCREEN1_6x12 | 134.00 | 47.50 | 10.00 | 0.00 | 1-#4 | SB | 5,527 | 5,264 | 0.76 | 0.61 |
| SCREEN2_6x12 | 134.00 | 47.50 | 10.00 | 0.00 | 1-#4 | SB | 5,364 | 5,190 | 0.72 | 0.58 |
| SCREEN3_6x24 | 134.00 | 48.00 | 21.50 | 0.00 | 1-#4 | SB | 13,732 | 11,006 | 0.66 | 0.22 |
| Short-Span ICF Lintel Specimens (previous testing) | | | | | | | | | | |
| FLAT1_4x12 | 43.00 | 14.33 | 10.00 | 4.00 | 1-#4 | S | 17,172 | - | 0.12 | - |
| FLAT2_4x12 | 43.00 | 14.33 | 10.00 | 4.00 | 1-#4 | S | 17,830 | - | 0.14 | - |
| FLAT1_4x24 | 43.00 | 14.33 | 22.00 | 4.00 | 1-#4 | S | 27,170 | - | 0.03 | - |
| FLAT1_8x12 | 43.00 | 14.33 | 10.00 | 8.00 | 1-#4 | S | 21,030 | - | 0.07 | - |
| FLAT2_8x12 | 43.00 | 14.33 | 10.00 | 8.00 | 1-#4 | S | 22,600 | - | 0.08 | - |
| FLAT1_8x24 | 43.00 | 14.33 | 22.00 | 8.00 | 1-#4 | S | 44,210 | - | 0.02 | - |
| FLAT1_4x12a | 40.00 | 13.33 | 10.00 | 4.00 | 2-#5 | - | N/A ⁵ | N/A ⁵ | N/A ⁵ | N/A ⁵ |
| FLAT1_8x12a | 40.00 | 13.33 | 10.00 | 8.00 | 2-#5 | S | 64,750 | - | 0.12 | - |
| WAFFLE1_6x8 | 36.00 | 12.00 | 6.00 | 2.00 | 1-#4 | S | 12,130 | - | 0.21 | - |
| WAFFLE2_6x8 | 36.00 | 12.00 | 6.00 | 2.00 | 1-#4 | S | 11,980 | - | 0.17 | - |
| WAFFLE1_6x16 | 36.00 | 12.00 | 14.00 | 2.00 | 1-#4 | S | 31,260 | - | 0.11 | - |
| WAFFLE2_6x16 | 36.00 | 12.00 | 14.00 | 2.00 | 1-#4 | S | 31,820 | - | 0.15 | - |
| WAFFLE1_8x16 | 36.00 | 12.00 | 13.75 | 2.00 | 1-#4 | S | 35,620 | - | 0.11 | - |
| WAFFLE2_8x16 | 36.00 | 12.00 | 13.75 | 2.00 | 1-#4 | S | 37,120 | - | 0.10 | - |
| SCREEN1_6x12 | 38.00 | 10.00 | 10.00 | 0.00 | 1-#4 | S | 6,498 | - | 0.29 | - |
| SCREEN2_6x12 | 38.00 | 10.00 | 10.00 | 0.00 | 1-#4 | S | 7,052 | - | 0.26 | - |
| SCREEN1_6x24 | 38.00 | 10.00 | 22.00 | 0.00 | 1-#4 | S | 30,460 | - | 0.14 | - |
| SCREEN2_6x24 | 38.00 | 10.00 | 22.00 | 0.00 | 1-#4 | S | 31,520 | - | 0.12 | - |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm; 1 lb = 4.45 N.

Notes:

- The shear span is defined as the distance from the support to the first load point.
- Failure Modes are as follows:
S: Shear before yield of tensile steel
SB: Shear after bending yield in tensile steel
BD: Excessive bending deflection due to yielding of tension steel (No shear failure).
- The measured deflection at the ultimate capacity of the lintel.
- The measured deflection at the yield point of the lintel (yield point of the reinforcing steel).
- A tested value of 16,750 lb was recorded. Premature failure was experienced due to severe honeycombing along the tension face caused by the two-#5 rebar which restricted the flow of the concrete to the bottom of the form.

Long-Span Flat Specimens

Failure of the long-span flat lintel specimens was characterized by a bending failure marked by yielding of the tensile reinforcing steel followed by excessive deflections (Figure 5a through 5c). Vertical cracks formed at midspan as the tensile reinforcing steel began to yield. Many of the vertical cracks occurred between the ties that connect the two sides of the form together. None of the long-span flat lintel specimens experienced diagonal tension cracks characterizing shear failures. The low tensile reinforcing steel ratios prevented large bending capacities from developing, hence reducing the shear stresses in the specimens.



Figure 5a
Typical Failure Mode for the Long-Span ICF Flat Lintel
(4x12 specimen after excessive deflection and tensile steel yielding)

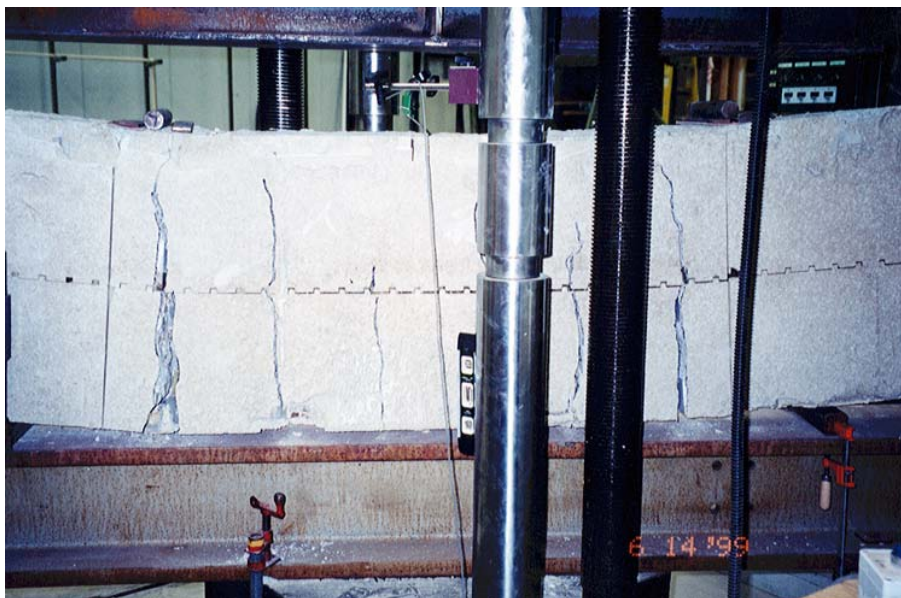


Figure 5b
Typical Failure Mode for the Long-Span ICF Flat Lintel
(4x24 specimen after excessive deflection and tensile steel yielding)



Figure 5c
Typical Failure Mode for the Long-Span ICF Flat Lintel
(4x24 specimen with foam removed at primary failure location)

Long-Span Waffle-Grid Specimens

Unlike the flat-wall lintels, the waffle-grid specimens ultimately failed in shear with characteristic diagonal tension cracks developing near the end reactions after significant yielding of the tensile reinforcing steel in the middle region of the lintel (Figure 6). The cracks that ultimately caused failure were web-shear cracks. Prior to shear failure, each beam experienced yielding of the tensile reinforcing steel and significant deflection. The large deflection at mid-span and gradual yielding of the tensile reinforcing steel prior to the shear related failure may be considered a safe failure mode since it was preceded by a ductile yielding of the tensile reinforcing steel as is intended in the *ACI 318* design standard.

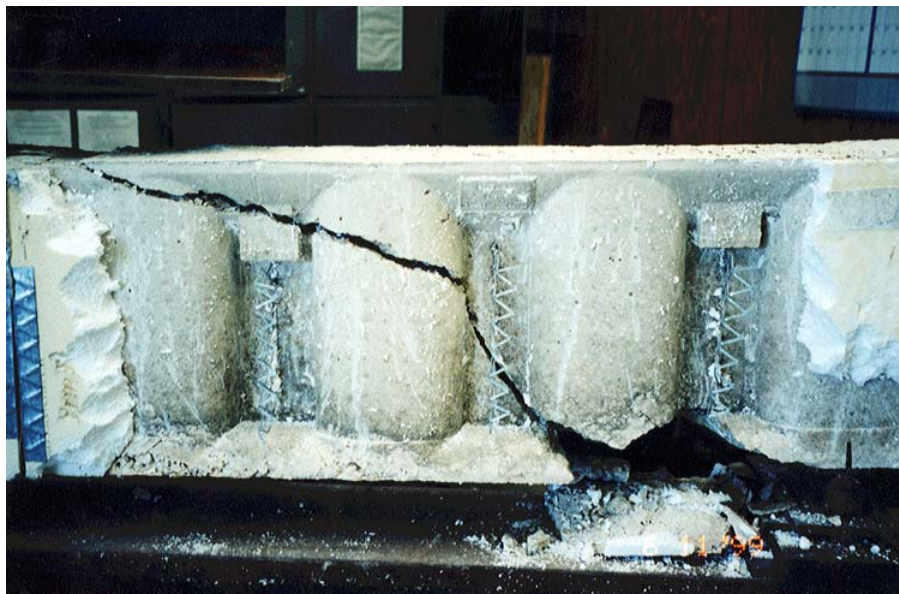


Figure 6
Typical Failure Mode for the Long-Span ICF Waffle-Grid Lintel

Long-Span Screen-Grid Specimens

The failure mode for the screen-grid systems was similar to that of the waffle-grids. The behavior was characterized by diagonal tension cracks that led to shear failure of the specimens. However, the specimens experienced yielding of the tensile reinforcing steel prior to the shear failure near the reaction point (Figures 7a & 7b). Although the deflections measured at ultimate capacity for the long-span screen-grid specimens were less than that experienced by the long-span flat and waffle-grid specimens, the measured deflections at mid-span and gradual yielding of the tensile reinforcing steel gave adequate warning of collapse and an initial ductile behavior. The amount of ductile steel yielding was somewhat less than that experienced by the waffle-grid and flat systems.



Figure 7a
Typical Failure Mode for the Long-Span ICF Screen-Grid Lintel
(6x12 specimen)



Figure 7b
Typical Failure Mode for the Long-Span ICF Screen-Grid Lintel
(6x24 specimen with foam removed from primary failure location)

Short-Span Lintel Specimens

The short-span lintel specimens from the earlier phase of testing experienced web-shear cracks with no evidence of flexural distress. The loads experienced were above that required for typical residential applications. Much of this capacity may be attributed to an arching effect in the short-span lintels that effectively transferred the applied load to the reactions. Small deflections and abrupt collapses characterized many of the failures. Although the failures were abrupt in nature, the tested loads were greater than that predicted by ACI Equation 11-3. A more detailed account of the testing can be found in the previous test report [1].

Design Evaluation

Many of the design issues involved in the analysis of concrete lintels were discussed in greater detail previously in this report. These included:

- (1) Minimum tensile steel reinforcement amounts for concrete lintels
- (2) Calculation of shear capacity of concrete lintels
- (3) Calculation of flexural capacity of concrete lintels

These three design issues are investigated in depth for both the short-span and long-span lintels in the following section and compared to the actual test results.

Flat Lintel Specimens

Table 6 summarizes the tensile reinforcing steel design conditions for the flat lintel specimens. The provision for a minimum amount of tensile reinforcing steel, $A_{s,min}$, in ACI 318-99 Equation 10-3 is intended to prevent sudden failures. Table 6 reveals that the actual area of tensile reinforcing steel in the test specimens ranged from 0.4 to 4.4 times $A_{s,min}$. This indicates that, in general, the amount of tensile reinforcing steel was less than that required by ACI 318.

Table 6
Comparison of Tensile Reinforcing Steel of Flat Lintel Specimens to
ACI 318-99 Minimum Requirements

| Test Specimen | Failure Mode ¹ | Reinforcement Ratio (ρ_w) | A_s^2 (in ²) | $A_{s,min}^3$ (in ²) | Ratio to $A_{s,min}$ ($A_s/A_{s,min}$) |
|--------------------------------|---------------------------|----------------------------------|----------------------------|----------------------------------|--|
| Long – Span ICF Lintel | | | | | |
| FLAT1 8x24 | BD | 0.00115 | 0.20 | 0.54 | 0.37 |
| FLAT2 4x12 | BD | 0.00500 | 0.20 | 0.12 | 1.67 |
| FLAT3 8x12 | BD | 0.00250 | 0.20 | 0.25 | 0.80 |
| FLAT4 4x24 | BD | 0.00227 | 0.20 | 0.27 | 0.74 |
| Short – Span ICF Lintel | | | | | |
| FLAT1 4x12 | S | 0.00500 | 0.20 | 0.12 | 1.67 |
| FLAT2 4x12 | S | 0.00500 | 0.20 | 0.12 | 1.67 |
| FLAT1 4x24 | S | 0.00227 | 0.20 | 0.27 | 0.74 |
| FLAT1 8x12 | S | 0.00250 | 0.20 | 0.25 | 0.80 |
| FLAT2 8x12 | S | 0.00250 | 0.20 | 0.25 | 0.80 |
| FLAT1 8x24 | S | 0.00114 | 0.20 | 0.54 | 0.37 |
| FLAT1 4x12a | - | 0.01525 | 0.61 | 0.14 | 4.36 |
| FLAT1 8x12a | S | 0.00763 | 0.61 | 0.27 | 2.26 |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm.

Notes:

1. Failure Modes are as follows:

S: Shear before yield of tensile steel

BD: Excessive bending deflection due to yielding of tension steel (No shear failure)

2. Actual area of tensile reinforcing steel.

3. Minimum area of tensile reinforcing steel required by ACI 318-99 Equation 10-3.

Table 7 summarizes the design calculations for both the shear and moment capacities in the flat ICF lintels. Equations 11-3, 11-5, and 11-29 from *ACI 318-99* were used to calculate the shear capacities of the short-span lintels. The moment capacity was not calculated for the short-span flat lintels since the specimens failed in shear. Similarly the shear capacity for the long-span flat lintels was not calculated since the specimens failed in bending.

Based on the calculations shown in Table 7, there does not appear to be any adverse or unpredictable effect related to the amount of tensile reinforcing steel in this test program and the failure mode (i.e. no sudden failures). Table 6 demonstrates that the minimum tensile reinforcing steel required for the types of beams tested could be reduced to $A_{s,min}$ equal to 0.20 in² (129.0 mm²) or that required by ACI 318-99 Equation 10-3, whichever is less.

However, the span length did affect the failure mode as expected. The long-span flat lintel specimens experienced a bending failure (characterized by large deflections and yielding of the tensile reinforcing steel) while the short-span lintels experienced shear failures (characterized by diagonal tension cracks).

Table 7
Tested vs. Predicted Shear and Moment Capacities Based on ACI 318-99
For the Flat ICF Lintels

| Test Specimen | Failure Mode ¹ | Tested Shear Capacity ² (lb) | Predicted Shear Capacity ³ (lb) | | | Ratio Tested/Predicted Shear | | | Tested Moment Capacity ⁴ (in-lb) | Predicted Moment Capacity ⁵ (in-lb) | Ratio Tested/Predicted Moment |
|--|---------------------------|---|--|-------------------|--------------------|------------------------------|-------------------|--------------------|---|--|-------------------------------|
| | | | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | | | |
| Long – Span ICF Lintel | | | | | | | | | | | |
| FLAT1_8x24 | BD | - | - | - | - | - | - | - | 293,558 | 277,278 | 1.06 |
| FLAT2_4x12 | BD | - | - | - | - | - | - | - | 116,823 | 121,066 | 0.96 |
| FLAT3_8x12 | BD | - | - | - | - | - | - | - | 130,204 | 125,233 | 1.04 |
| FLAT4_4x24 | BD | - | - | - | - | - | - | - | 269,859 | 276,347 | 0.98 |
| Short – Span ICF Lintel⁶ | | | | | | | | | | | |
| FLAT1_4x12 | S | 8,586 | 4,568 | 4,879 | 8,700 | 1.88 | 1.76 | 1.07 | - | - | - |
| FLAT2_4x12 | S | 8,915 | 4,568 | 4,879 | 8,700 | 1.95 | 1.83 | 1.02 | - | - | - |
| FLAT1_4x24 | S | 13,585 | 10,049 | 10,087 | 28,011 | 1.35 | 1.35 | 0.48 | - | - | - |
| FLAT1_8x12 | S | 10,515 | 9,136 | 9,219 | 16,113 | 1.15 | 1.14 | 0.65 | - | - | - |
| FLAT2_8x12 | S | 11,300 | 9,136 | 9,219 | 16,113 | 1.24 | 1.23 | 0.70 | - | - | - |
| FLAT1_8x24 | S | 22,105 | 20,098 | 19,633 | 51,877 | 1.10 | 1.13 | 0.43 | - | - | - |
| FLAT1_4x12a ^{7,8} | S | - | 4,568 | - | - | - | - | - | - | - | - |
| FLAT1_8x12a ⁷ | S | 32,375 | 9,136 | 10,326 | 20,440 | 3.54 | 3.14 | 1.58 | - | - | - |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm; 1 lb = 4.45 N, 1 in-lb = 0.113 N-m

Notes:

- Failure Modes are as follows:
 S: Shear before yield of tensile steel
 SB: Shear after bending yield in tensile steel
 BD: Excessive bending deflection due to yielding of tension steel (No shear failure)
- The tested shear capacity is equal to one-half the ultimate recorded load.
- The predicted shear capacities are adjusted to represent a mean prediction.
- Tested moment capacity was determined from the yield point.
- Predicted moment calculations are derived from ACI Chapter 10.
- All specimens meet the deep beam classification since $l_w/h < 5$.
- Specimens contained 2-#5 tensile reinforcing bars (the other specimens contained 1-#4 steel reinforcing bar).
- A tested value of 16,750 lb was recorded. Premature failure was experienced due to severe honeycombing on the tension face caused by the two-#5 rebar which restricted the flow of the concrete to the bottom of the form.

The predicted bending capacity provides an accurate estimate for the long-span flat ICF lintels. The ratios of tested to predicted moment capacity range from 0.96 to 1.06. The bending capacity for the short-span ICF lintels was not determined since all of the specimens experienced a shear failure.

As shown in Table 7, the shear capacity is conservatively predicted using ACI Equation 11-3 or 11-5, while ACI Equation 11-29 predictions are generally less than conservative. The data also indicates that ACI Equation 11-3 and 11-5 produce very similar predictions for the specimens tested, while Equation 11-29 estimates much larger capacities when the lintel qualifies as a deep beam according to *ACI 318*. The predicted shear capacities using Equation 11-3, 11-5, and 11-29 in Table 7 were adjusted to mean estimates by multiplying the calculated values by

$$[1 - (\Phi^{-1}(z)) * COV] = [1 - (\Phi^{-1}(0.30)) * (0.151)] = [1 - (-0.52)(0.151)] = 1.08.$$

The above adjustment was done to provide relevant comparisons to the test data since the ACI Equations provide a 30-percentile estimate as mentioned earlier. A normal distribution is assumed in the derivation of the adjustment.

Figure 8 indicates that the reinforcement ratio, ρ_w (or A_s/b_wd), has a significant impact on the actual shear capacity of the flat lintel specimens for both ACI Equation 11-5 and 11-29. Ratios greater than 1.0 indicate a conservative prediction. Although ACI Equation 11-5 and 11-29 includes ρ_w in the calculation of the shear capacity, Figure 8 demonstrates that ρ_w should play a larger role in the calculation. As the reinforcement ratio increases, so does the ratio of tested to predicted shear capacity. ACI Equation 11-29 also includes the additional increase for small span-to-depth ratios.

While the amount of tensile steel present does not have an adverse or unpredictable effect related to the failure mode it, does seem to affect the predictions computed using *ACI 318-99*. Figure 8 indicates that ACI Equation 11-5 conservatively predicts the shear capacity for the concrete beams tested regardless of reinforcement ratio or span-to-depth ratio. However, as the tensile reinforcing steel ratio increases the predicted shear capacity becomes increasingly conservative. The deep beam provisions of ACI Equation 11-29 (span-to-depth ratio, l_n/h , less than 5) does not similarly provide conservative predictions of shear capacity for all of the beams tested. Specimens that met or exceeded the minimum tensile steel requirements outlined by *ACI 318-99*, denoted by the ratio of A_s to $A_{s,min}$ greater than 1.0 in Table 6, are associated with conservative estimates of shear capacity while those not meeting the minimum tensile steel requirements resulted in overestimated shear capacities.

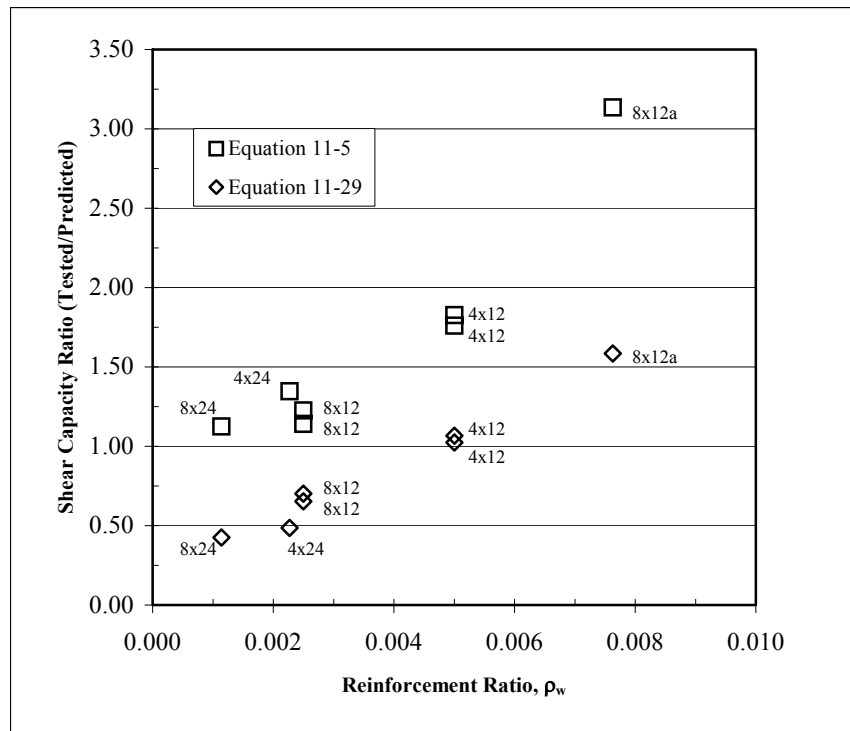


Figure 8
The Influence of the Reinforcement Ratio on the Predicted Shear Capacity for the Short-Span Flat ICF Lintel Specimens

Waffle-Grid Specimens

Table 8 summarizes the tensile steel reinforcement design conditions for the waffle-grid lintel specimens. The values for $A_{s,min}$ are significantly less than that calculated for the flat lintel specimens because the thickness of the web, b_w , is 2 in (50.8 mm) for both the 6 in (101.6 mm) and the 8 in (152.4 mm) waffle-grid specimens. Table 8 reveals that the actual area of tensile reinforcing steel ranges from 2.2 to 5.0 times the ACI required $A_{s,min}$ (based on the web thickness, b_w , and beam depth, d).

Table 8
Comparison of Tensile Reinforcing Steel of Waffle-Grid Lintel Specimens to
ACI 318-99 Minimum Requirements and to Balanced Conditions

| Test Specimen | Failure Mode ¹ | Reinforcement Ratio (ρ_w) | A_s^2 (in ²) | $A_{s,min}^3$ (in ²) | Ratio to $A_{s,min}$ ($A_s/A_{s,min}$) |
|--------------------------------|---------------------------|----------------------------------|----------------------------|----------------------------------|--|
| Long – Span ICF Lintel | | | | | |
| WAFFLE1 8x16 | SB | 0.00714 | 0.20 | 0.09 | 2.22 |
| WAFFLE2 8x16 | SB | 0.00741 | 0.20 | 0.08 | 2.50 |
| WAFFLE3 8x16 | SB | 0.01429 | 0.40 | 0.09 | 4.44 |
| WAFFLE4 8x8 | SB | 0.01667 | 0.20 | 0.04 | 5.00 |
| Short – Span ICF Lintel | | | | | |
| WAFFLE1 6x8 | S | 0.01667 | 0.20 | 0.04 | 5.00 |
| WAFFLE2 6x8 | S | 0.01667 | 0.20 | 0.04 | 5.00 |
| WAFFLE1 6x16 | S | 0.00714 | 0.20 | 0.09 | 2.22 |
| WAFFLE2 6x16 | S | 0.00714 | 0.20 | 0.09 | 2.22 |
| WAFFLE1 8x16 | S | 0.00727 | 0.20 | 0.09 | 2.22 |
| WAFFLE2 8x16 | S | 0.00727 | 0.20 | 0.09 | 2.22 |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm.

Notes:

- Failure Modes are as follows:
 S: Shear before yield of tensile steel
 SB: Shear after bending yield in tensile steel
- Actual area of tensile reinforcing steel.
- Minimum area of tensile reinforcing steel required by ACI 318-99 Equation 10-3.

Table 9 summarizes the design calculations for both the shear and moment capacities for the waffle-grid ICF lintels. Both the shear and moment capacities for the long-span waffle-grid lintels were calculated since the specimens first experienced yielding of the tensile reinforcing steel before ultimately failing in shear. The moment capacity was not calculated for the short-span waffle-grid lintels since the specimens failed in shear.

As shown in Table 9, the shear capacity is conservatively predicted using ACI Equation 11-3, 11-5, or 11-29. The predicted shear capacities using Equation 11-3, 11-5, and 11-29 in Table 9 were adjusted to mean estimates by multiplying the calculated values by

$$[1 - (\Phi^{-1}(z)) * COV] = [1 - (\Phi^{-1}(0.30)) * (0.151)] = [1 - (-0.52)(0.151)] = 1.08.$$

The above adjustment was done to provide relevant comparisons to the test data since the ACI Equations provide a 30-percentile estimate as mentioned earlier. A normal distribution is assumed in the derivation of the adjustment.

The ratio of tested to predicted shear capacity in Table 9 indicates that the shear capacity of the waffle-grid specimens is conservatively predicted using ACI Equations 11-3 or 11-5. When applicable, ACI Equation 11-29 for deep beams also conservatively predicts the shear capacity of the waffle-grid concrete lintels that qualify as deep beams. ACI Equations 11-3 and 11-5 produce very different shear capacity predictions for the waffle-grid specimens tested. The difference in predictions may be attributed to the contribution of the tensile reinforcing steel ratio in Equation 11-5 since this value is substantially larger in the waffle-grid lintels than that calculated in the flat wall lintels (due to decreased web thickness).

ACI Equation 11-5 produces less conservative shear capacity predictions for the waffle-grid lintels compared to that of ACI Equation 11-3. The predicted bending capacity is accurately predicted for the waffle-grid long-span ICF lintels. The predicted bending capacity is 0.98 to 1.06 times less than the actual bending capacity.

Table 9
Tested vs. Predicted Shear and Moment Capacities Based on ACI 318-99
For the Waffle-Grid ICF Lintels

| Test Specimen | Failure Mode ¹ | Tested Shear Capacity ² (lb) | Predicted Shear Capacity ³ (lb) | | | Ratio Tested/Predicted Shear | | | Tested Moment Capacity ⁴ (in-lb) | Predicted Moment Capacity ⁵ (in-lb) | Ratio Tested/Predicted Moment |
|--------------------------------|---------------------------|---|--|-------------------|--------------------|------------------------------|-------------------|--------------------|---|--|-------------------------------|
| | | | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | | | |
| Long – Span ICF Lintel | | | | | | | | | | | |
| WAFFLE1_8x16 | SB | 5,058 | 3,170 | 3,552 | - | 1.44 | 1.24 | | 165,888 | 169,928 | 0.98 |
| WAFFLE2_8x16 | SB | 5,261 | 3,288 | 3,663 | - | 1.60 | 1.48 | | 181,272 | 176,398 | 1.03 |
| WAFFLE3_8x16 ⁶ | SB | 8,881 | 3,288 | 4,203 | - | 2.70 | 2.11 | | 363,688 | 343,272 | 1.06 |
| WAFFLE4_8x8 | SB | 1,841 | 1,409 | 1,879 | - | 1.31 | 0.98 | | 77,208 | 72,878 | 1.06 |
| Short – Span ICF Lintel | | | | | | | | | | | |
| WAFFLE1_6x8 | S | 6,065 | 1,370 | 1,842 | - | 4.43 | 3.29 | - | - | - | - |
| WAFFLE2_6x8 | S | 5,990 | 1,370 | 1,842 | - | 4.37 | 3.25 | - | - | - | - |
| WAFFLE1_6x16 ⁶ | S | 15,630 | 3,197 | 3,578 | 9,592 | 4.89 | 4.37 | 1.63 | - | - | - |
| WAFFLE2_6x16 ⁶ | S | 15,910 | 3,197 | 3,578 | 9,592 | 4.98 | 4.45 | 1.66 | - | - | - |
| WAFFLE1_8x16 ⁶ | S | 17,810 | 3,140 | 3,523 | 9,421 | 5.67 | 5.05 | 1.89 | - | - | - |
| WAFFLE2_8x16 ⁶ | S | 18,560 | 3,140 | 3,523 | 9,421 | 5.91 | 5.27 | 1.97 | - | - | - |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm; 1 lb = 4.45 N, 1 in-lb = 0.113 N-m

Notes:

1. Failure Modes are as follows:
 S: Shear before yield of tensile steel
 SB: Shear after bending yield in tensile steel
2. The tested shear capacity is equal to one-half the ultimate recorded load.
3. The predicted shear capacities are adjusted to represent mean predictions.
4. Tested moment capacity was determined from the yield point
5. Predicted moment calculations are derived from ACI Chapter 10.
6. Specimens contained 2-#5 tensile reinforcing bars and the remaining specimens contained 1-#4 steel reinforcing bar.
7. Meets deep beam classification since $l_w/h < 5$.

The ratio of tested to predicted shear capacity for ACI Equation 11-3 and 11-29 are considerably larger than those computed for the flat lintel specimens because the waffle-grid specimens are conservatively assumed to be a rectangular member with a uniform width equivalent to the thickness of the web, b_w . Table 10 shows an effective web thickness for the waffle-grid specimens that produce exact predictions using the ACI Equation 11-3 and 11-29. A conservative effective web thickness of 2.6 in (66.0 mm) can be used in ACI Equation 11-3 and an effective web thickness of 3.2 in (81.3 mm) can be used in ACI Equation 11-29 (when applicable, $l_n/h < 5$) for computing the shear capacity of the waffle-grid lintels. ACI Equation 11-5 can be used with the actual web thickness of 2.0 in (50.8 mm) as shown to be suitable in the long-span lintels of Table 9.

Table 10
Equivalent Web Thickness for Waffle-Grid Specimens

| Test Specimen | Tested Shear Capacity ¹ (lb) | Predicted Shear Capacity ² (lb) | | Effective Width, b_w (in) | |
|--------------------------------|---|--|--------------------|-----------------------------|--------------------|
| | | ACI Equation 11-3 | ACI Equation 11-29 | ACI Equation 11-3 | ACI Equation 11-29 |
| Long – Span ICF Lintel | | | | | |
| WAFFLE1_8x16 | 5,058 | 3,170 | - | 3.19 | - |
| WAFFLE2_8x16 | 5,261 | 3,288 | - | 3.20 | - |
| WAFFLE3_8x16 ⁵ | 8,881 | 3,288 | - | 5.40 | - |
| WAFFLE4_8x8 | 1,841 | 1,409 | - | 2.61 | - |
| Short – Span ICF Lintel | | | | | |
| WAFFLE1_6x8 | 6,065 | 1,370 | - | 8.85 | - |
| WAFFLE2_6x8 | 5,990 | 1,370 | - | 8.74 | - |
| WAFFLE1_6x16 | 15,630 | 3,197 | 9,592 | 9.77 | 3.26 |
| WAFFLE2_6x16 | 15,910 | 3,197 | 9,592 | 9.95 | 3.32 |
| WAFFLE1_8x16 | 17,810 | 3,140 | 9,421 | 11.34 | 3.78 |
| WAFFLE2_8x16 | 18,560 | 3,140 | 9,421 | 11.82 | 3.94 |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm; 1 lb = 4.45 N.

- Notes: 1. The tested shear capacity is equal to one-half the ultimate recorded load.
2. The predicted shear capacities are adjusted to represent mean predictions.

Screen-Grid Specimens

Table 11 summarizes the tensile steel reinforcement design conditions for the screen-grid lintel specimens. There is no minimum steel reinforcement requirement because the screen-grid has a non-solid cross-section, therefore, the thickness of the web, b_w , is zero.

Table 11
Comparison of Tensile Reinforcing Steel of Screen-Grid Lintel Specimens to
ACI 318-99 Minimum Requirements and to Balanced Conditions

| Test Specimen | Failure Mode ¹ | Reinforcement Ratio (ρ_w) | A_s^2 (in ²) | $A_{s,min}^3$ (in ²) | Ratio to $A_{s,min}$ ($A_s/A_{s,min}$) |
|--------------------------------|---------------------------|----------------------------------|----------------------------|----------------------------------|--|
| Long – Span ICF Lintel | | | | | |
| SCREEN1 6x12 | SB | - | 0.20 | 0 ⁴ | - |
| SCREEN2 6x12 | SB | - | 0.20 | 0 ⁴ | - |
| SCREEN3 6x24 | SB | - | 0.20 | 0 ⁴ | - |
| Short – Span ICF Lintel | | | | | |
| SCREEN1 6x12 | S | - | 0.20 | 0 ⁴ | - |
| SCREEN2 6x12 | S | - | 0.20 | 0 ⁴ | - |
| SCREEN1 6x24 | S | - | 0.20 | 0 ⁴ | - |
| SCREEN2 6x24 | S | - | 0.20 | 0 ⁴ | - |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm.

Notes:

1. Failure Modes are as follows:
 - S: Shear before yield of tensile steel
 - SB: Shear after bending yield in tensile steel
2. Actual area of tensile reinforcing steel.
3. Minimum area of tensile reinforcing steel required by ACI 318-99 Equation 10-3.
4. The minimum tensile steel reinforcement required is zero because $b_w = 0$.

Table 12 summarizes the design calculations for both the shear and moment capacities for the screen-grid ICF lintels. The moment capacity was not calculated for the short-span screen-grid lintels since the specimens failed in shear. Both the shear and moment capacities for the long-span screen-grid lintels were calculated since the specimens first experienced yielding of the tensile reinforcing steel, but ultimately failed in shear. The testing indicates that the screen-grid lintels can sustain significant shear loads despite the absence of a method to analyze such lintels in *ACI*

318-99. There does not appear to be any adverse affect associated with the amount of tensile reinforcing steel as presented in Table 11 and the failure mode. As with the flat and waffle grid lintels, the predicted bending capacity is also accurately estimated for the screen-grid long-span ICF lintels. The tested bending capacity is 0.97 to 1.01 that of the predicted bending capacity.

Table 12
Tested vs. Predicted Shear and Moment Capacities Based on ACI 318-99
For the Screen-Grid ICF Lintels

| Test Specimen | Failure Mode ¹ | Tested Shear Capacity ² (lb) | Predicted Shear Capacity ³ (lb) | | | Ratio Tested/Predicted Shear | | | Tested Moment Capacity ⁴ (in-lb) | Predicted Moment Capacity ⁵ (in-lb) | Ratio Tested/Predicted Moment |
|--|---------------------------|---|--|-------------------|--------------------|------------------------------|-------------------|--------------------|---|--|-------------------------------|
| | | | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | | | |
| Long – Span ICF Lintel | | | | | | | | | | | |
| SCREEN1_6x12 | SB | 2,764 | 0 | 0 | - | - | - | - | 125,020 | 123,344 | 1.01 |
| SCREEN2_6x12 | SB | 2,682 | 0 | 0 | - | - | - | - | 123,263 | 123,344 | 1.00 |
| SCREEN3_6x24 | SB | 6,866 | 0 | 0 | - | - | - | - | 264,144 | 272,154 | 0.97 |
| Short – Span ICF Lintel⁶ | | | | | | | | | | | |
| SCREEN1_6x12 | S | 3,249 | 0 | 0 | 0 | - | - | - | - | - | - |
| SCREEN2_6x12 | S | 3,526 | 0 | 0 | 0 | - | - | - | - | - | - |
| SCREEN1_6x24 | S | 15,230 | 0 | 0 | 0 | - | - | - | - | - | - |
| SCREEN2_6x24 | S | 15,760 | 0 | 0 | 0 | - | - | - | - | - | - |

For SI: 1 ft = 0.30 m; 1 in = 25.4 mm; 1 lb = 4.45 N, 1 in-lb = 0.113 N-m

Notes:

1. Failure Modes are as follows:

S: Shear before yield of tensile steel

SB: Shear after bending yield in tensile steel

2. The tested shear capacity is equal to one-half the ultimate recorded load.

3. No method exists to predict shear capacity of screen-grid ICF lintels in ACI 318.

4. Tested moment capacity was determined from the yield point

5. Predicted moment calculations are derived from ACI Chapter 10.

6. Meets deep beam classification since $l_w/h < 5$.

Table 13 computes an equivalent web thickness for the screen-grid specimens using ACI Equations 11-3, 11-5, and 11-29. A conservative effective web thickness of 2.2 in. (55.9 mm) can be used in ACI Equation 11-3 and 11-5 when computing the shear capacity of the screen-grid lintels. When applicable, an effective web thickness of 0.9 in (22.9 mm) can be used in ACI Equation 11-29 for computing the shear capacity of 12 in (304.8 mm) deep the screen-grid lintels meeting the deep beam requirements. When the lintel depth is 24 in (609.6 mm) or greater, an effective width of 2.0 in (50.8 mm) is applicable.

Table 13
Equivalent Web Thickness for Screen-Grid Specimens

| Test Specimen | Tested Shear Capacity ¹ (lb) | Predicted Shear Capacity ² (lb) | | | Effective Width, b_w (in) | | |
|--------------------------------|---|--|-------------------|--------------------|-----------------------------|-------------------|--------------------|
| | | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 | ACI Equation 11-3 | ACI Equation 11-5 | ACI Equation 11-29 |
| Long – Span ICF Lintel | | | | | | | |
| SCREEN1_6x12 | 2,764 | 0 | 0 | - | 2.35 | 2.28 | - |
| SCREEN2_6x12 | 2,682 | 0 | 0 | - | 2.28 | 2.21 | - |
| SCREEN3_6x24 | 6,866 | 0 | 0 | - | 2.72 | 2.75 | - |
| Short – Span ICF Lintel | | | | | | | |
| SCREEN1_6x12 | 3,249 | 0 | 0 | 0 | 2.85 | 2.50 | 0.95 |
| SCREEN2_6x12 | 3,526 | 0 | 0 | 0 | 3.09 | 2.75 | 1.03 |
| SCREEN1_6x24 | 15,230 | 0 | 0 | 0 | 6.06 | 6.15 | 2.02 |
| SCREEN2_6x24 | 15,760 | 0 | 0 | 0 | 6.27 | 6.38 | 2.09 |

For SI: 1 ft = 0.3048 m; 1 in = 25.4 mm; 1 lb = 4.45 N.

- Notes: 1. The tested shear capacity is equal to one-half the ultimate recorded load.
2. The predicted shear capacities are adjusted to represent mean predictions.

Design Recommendations

The following section provides summarized lintel design recommendations that incorporate the findings of this study for each ICF lintel configuration type. The following design recommendations apply to ICF lintels modeled with simply supported end conditions.

Flat ICF Lintels

- (1) Require a minimum amount of tensile reinforcing steel, $A_{s,min}$, determined by the lesser of:

- $A_{s,min} = 0.2 \text{ in}^2 \text{ (129.0 mm}^2\text{)}$
- $A_{s,min} = \frac{3\sqrt{f'_c}b_w d}{f_y} \geq \frac{200b_w d}{f_y}$ ACI Equation 10-3

- (2) If the area of tensile reinforcing steel, A_s , is greater than the minimum calculated using ACI 318 Equation 10-3 and the beam has a span-to-depth ratio (l_n/h) of less than 5, calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29. Use ACI Equations 11-3 or 11-5 to determine the shear capacity for all other scenarios.
- (3) Use shear reinforcement (stirrups) as required by *ACI 318-99*. This study was unable to draw conclusive recommendations regarding any change to limits for shear reinforcement requirements, particularly when shear reinforcement is necessary in longer span members.

Waffle-Grid ICF Lintels

- (1) Require a minimum amount of tensile reinforcing steel, $A_{s,min}$, determined by the lesser of:

- $A_{s,min} = 0.2 \text{ in}^2 \text{ (129.0 mm}^2\text{)}$
- $A_{s,min} = \frac{3\sqrt{f'_c}b_w d}{f_y} \geq \frac{200b_w d}{f_y}$ ACI Equation 10-3

- (2) If the beam has a span-to-depth ratio (l_n/h) of less than 5, calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29, with an effective web thickness of 3.2 in (81.3 mm).

If the beam has a span-to-depth ratio (l_n/h) of greater than 5, calculate the shear capacity with an effective web thickness of 2.6 in (66.0 mm) using ACI 318 Equation 11-3 or an effective web thickness of 2.0 in (50.8 mm) in ACI Equation 11-5.

- (3) Use shear reinforcement (stirrups) as required by *ACI 318-99*. This study was unable to draw conclusive recommendations regarding any change to limits for shear reinforcement requirements, particularly when shear reinforcement is necessary in longer span members.

Screen-Grid ICF Lintels

- (1) Require a minimum amount of tensile reinforcing steel, $A_{s,min}$, determined by the lesser of:

- $A_{s,min} = 0.2 \text{ in}^2 \text{ (129.0 mm}^2\text{)}$

- $A_{s,min} = \frac{3\sqrt{f'_c}b_w d}{f_y} \geq \frac{200b_w d}{f_y}$ ACI Equation 10-3

- (2) If the beam has a span-to-depth ratio (l_n/h) of less than 5, calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29, with an effective web thickness of 2.0 in (81.3 mm) for lintels with depths 24 in (609.6 mm) or greater, and an effective web thickness of 0.9 in (22.9 mm) for lintels with depths less than 24 in (609.6 mm).

If the beam has a span-to-depth ratio (l_n/h) of greater than 5, calculate the shear capacity with an effective web thickness of 2.2 in (55.8 mm) using ACI 318 Equation 11-3 or ACI Equation 11-5.

- (3) Use shear reinforcement (stirrups) as required by *ACI 318-99*. This study was unable to draw conclusive recommendations regarding any change to limits for shear reinforcement requirements, particularly when shear reinforcement is necessary in longer span members.

Conclusions

The purpose of this research program was to further evaluate the effectiveness of concrete lintels without shear reinforcement and produce more efficient lintel designs for ICF construction. The previous research study in May 1998 concluded that shear reinforcement was not necessary for ICF lintels spanning up to 4 ft (1.23 m). Although this recommendation may hold true for many ICF lintels spanning less than 4 ft (1.23 m), the aforementioned design recommendations in this report provide a more complete tool for analyzing all lintels. This study focused on the performance of long-span lintels along with a more detailed analysis of the short-span lintels tested in the previous report. All lintels tested in the second phase of this study were approximately 12 ft (3.69 m) in length. In all cases, the tested bending moments of the long-span ICF lintels without shear reinforcement were accurately predicted using *ACI 318-99*. The ratio of tested to predicted bending capacities ranged between 0.96 and 1.06, with values equal to 1.0 being an exact prediction.

All three types of the long-span ICF lintels tested (flat, waffle-grid, and screen-grid) experienced a yielding of the tensile reinforcing steel before failure. Although the waffle-grid and screen-grid lintels experienced an ultimate shear failure characterized by diagonal tension cracks, these failures occurred after the tensile reinforcing steel experienced significant yielding. The range of tensile reinforcing steel (i.e. reinforcement ratio, ρ_w) did not affect the performance of the lintels tested.

Assessments of the *ACI 318-99* shear design equations and proposed changes to the effective web thicknesses for the waffle-grid and screen grid lintels were developed to be consistent with the test data. In addition, design recommendations were outlined for the three ICF systems investigated.

This testing program reveals that ACI Equations 11-3 and 11-5 provide conservative predictions of shear capacity for all of the concrete lintels tested. When applicable the deep beam provisions of ACI Equation 11-29 provide a better estimate of the shear capacity. Certain limits to the use of these equations were explicitly defined in the proposed design approaches for the three ICF lintels evaluated. The two variables that have the greatest impact on shear capacity are the reinforcement ratio (ρ_w) and the span-to-depth ratio (l_n/h). The proposed design recommendations outlined in the previous section provide the design professional with additional tools in designing concrete lintels. These recommendations will also be utilized in future editions of the *Prescriptive Method*.

Recommendations

Additional testing and analysis should be done to expand on the findings of this study. Future topics of study should include:

- Additional lintel testing of long-span beams with sufficient tensile reinforcing steel to initiate shear failure prior to bending yield should be investigated.
- In addition, the beam end conditions in the current test program do not adequately simulate the “fixed-end” conditions and combined moment and shear that would exist in actual practice. Therefore, additional tests of beams under high shear and moment (but without shear reinforcement) should also be investigated using various reinforcement ratios.
- Conduct more sophisticated analysis, such as finite element analysis, on the waffle-grid and screen grid ICF lintel type to further examine the behavior of lintels with non-uniform web thicknesses to better understand behavior and to develop improved predictions based on the effects of ρ_w on the ACI shear equations.

References

- [1] *Lintel Testing for Reduced Shear Reinforcement in Insulating Concrete Form Systems*, Prepared for the U.S. Department of Housing and Urban Development, the Portland Cement Association, and the National Association of Home Builders by the NAHB Research Center, Inc., Upper Marlboro, Maryland, May 1998.

- [2] *Building Code Requirements for Reinforced Concrete (ACI 318-99)*, American Concrete Institute, Detroit, Michigan, 1999.
- [3] *Prescriptive Method for Insulating Concrete Forms in Residential Construction*, Prepared for the U.S. Department of Housing and Urban Development, the Portland Cement Association, and the National Association of Home Builders by the NAHB Research Center, Inc., Upper Marlboro, Maryland, 1998.
- [4] *The Shear Strength of Reinforced Concrete Members*, ASCE-ACI Task Committee 426, Proceedings of the American Society of Civil Engineers, Vol. 99, No. ST6, June 1973, pp. 1091-1187.
- [5] Nilson, A.H. and Winter, G., *Design of Concrete Structures*, McGraw-Hill, Inc. New York, New York, 1991.
- [6] *Shear and Diagonal Tension*, pt. 2, ACI-ASCE Committee 326, ACI Journal, Vol. 59, No. 2, February 1962, pp. 277-333.
- [7] *Suggested Revisions to Shear Provisions in Building Codes*, ACI-ASCE Committee 426, ACI Journal, Vol. 74, No. 9, September, 1977, pp. 458-469.
- [8] Kani, G., *Basic Facts Concerning Shear Failure*, American Concrete Institute, ACI Journal, Volume 63, No. 6, Detroit, Michigan, 1966.
- [9] *Structural Design of Insulating Concrete Form Walls in Residential Construction*, Prepared by the NAHB Research Center, Inc., Upper Marlboro, Maryland, 1998.
- [10] ASTM C143-96 *Slump of Portland Cement Concrete*, American Society of Testing Standards (ASTM), West Conshohocken, Pennsylvania, 1996.
- [11] ASTM C39-96 *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*, American Society of Testing Standards (ASTM), West Conshohocken, Pennsylvania, 1996.
- [12] ASTM C31-96 *Standard Practice for Making and Curing Concrete Test Specimens in the Field*, American Society of Testing Standards (ASTM), West Conshohocken, Pennsylvania, 1996.
- [13] ASTM C192-96 *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*, American Society of Testing Standards (ASTM), West Conshohocken, Pennsylvania, 1996.
- [14] ASTM C78-94 *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*, American Society of Testing Standards (ASTM), West Conshohocken, Pennsylvania, 1984.